The REMR Bulletin

News from the Repair, Evaluation, Maintenance,
and Rehabilitation Research Program

Volume 9, Number 1 March 1992

REMR Management Systems Training for US Army Corps of Engineers Personnel

by
David T. McKay

US Army Construction Engineering Research Laboratory

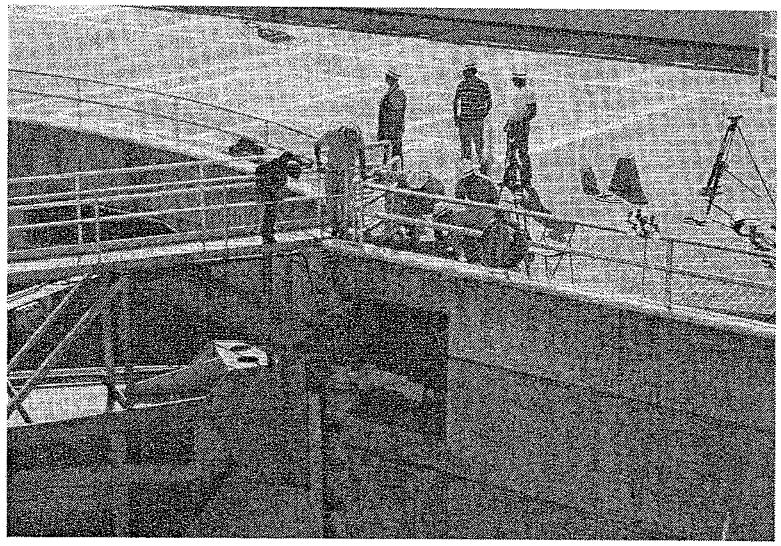


Figure 1. Inspection being made at site during training session.

Cost-effective maintenance and repair (M&R) planning for Civil Works structures requires information about their current condition, and the REMR Management Systems provide the decision support tool needed to make such an assessment. The microcomputer-based systems blend condition data with known M&R alternatives. This blend, coupled with life cycle cost analyses, allows consequence modeling to be performed so that various M&R strategies can be examined.

The concept of objective condition assessment is central to the REMR Management Systems. Standard field inspection procedures (Figure 1) yield data that in turn produce a Condition Index (CI), a numerical rating that ranges from 0 to 100 and is designed to capture a "snapshot" of a structure's current condition. The CI conforms to an established condition rating scale. The creation of the CI allows a global comparison of condition in similar structures. Continued use and storage of CI data over time provide the means to track trends in deterioration and predict future condition (Kao 1991).

Featured in This Issue

Training sessions

To ensure the validity and reliability of data obtained, the inspectors need to be trained in the procedures used in the field and in the basics for data entry. To date, four training sessions have been conducted for US Army Corps of Engineers (USACE) personnel.

The four USACE Divisions participating in REMR Management

Systems training thus far are North Pacific Division, The Dalles, July 1990; North Central Division, Lock and Dam No. 15 - Rock Island, April 1991; Ohio River Division, the Nashville Repair Station, August 1991; and Lower Mississippi Valley Division, Inner Harbor Navigation Canal Lock, December 1991. Each training site selected had close classroom facilities and was near a lock structure that had concrete lockwalls, miter gates, and steel sheet pile.

The general approach to the training exercise is in three phases. First, inspection procedures are explained in a classroom presentation.

Second, weather permitting, a CI inspection is performed during a field trip to the site (Figure 2). Due to time constraints, this inspection is abbreviated but sufficient to teach the techniques involved.

Third, participants return to the classroom to enter the data into computers and generate reports (Figure 3). It should be noted that the participants need not be computer literate, since the software is menu driven and user friendly.

Class sizes have ranged from 15 to 20 people, being composed of 3 or more people from each District within the given Division. Each District has been represented by people with a variety of backgrounds and job responsibilities.

Typical agenda

The session lasts 5 working days, with each system being covered in a series, starting with the concrete lockwall, then the miter gate, and finally the steel sheet-pile system. Beginning early Monday afternoon, a general overview of the REMR Management Systems is presented, followed by classroom instruction for the concrete lockwall system. The class adjourns in the late afternoon and meets again Tuesday morning at the lock site to perform the CI inspection. The afternoon session is spent in the classroom entering the data obtained into the system and generating CI reports. Beginning Wednesday morning, the same three-step procedure is used for the miter gate, with data entry being completed early Thursday afternoon. Instruction on the steel sheet-pile system extends from Thursday afternoon through Friday morning, leaving travel time on Friday afternoon.

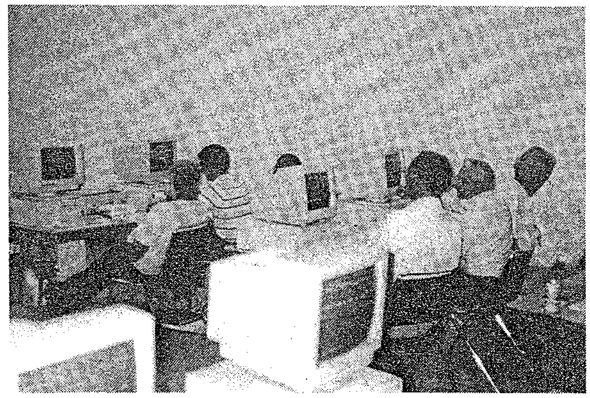


Figure 3. Training participants entering CI inspection data into REMR Management System software.

All training sessions to date have proceeded very smoothly with only occasional and minor operational hindrances. Some skepticism as to the usefulness of the system has been encountered, but the consensus at the end of the training is that the CIs obtained are indeed meaningful and useful. Some diversity in the CIs obtained from the various inspectors was noted, but deemed acceptable, and attributed to the fact that it was a first time experience for all involved. With practice, an inspection team of two or more people can do an entire CI inspection, complete with data entry, in 2 days.

Some inspection details

Equipment requirements for an inspection include a transit, level, rod, rulers, dial gages, feeler gages, tape measures, and C-clamps. Support may be required from the lock-operating personnel for minor shop fabrications. A boat and operator are needed for each inspection. Lock traffic may have to be stopped for up to an hour, but such an occurrence is rare.

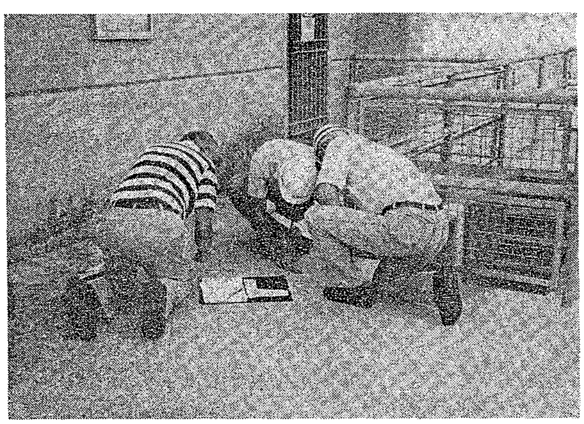


Figure 2. Inspectors measuring the width of a crack in a concrete lockwall monolith.

The concrete lockwall inspection entails both walking and boat tours at high and low pool. Crack widths, location, and orientation are recorded. Losses of volume because of deterioration are measured. The extent of spalled joints, leaks, and damaged or missing armor is noted.

All miter gate observations are made from the deck, the gate anchorage recess, or a boat. Noise, unusual vibration, misalignment, or changes in elevation during gate operation are noted. The existence of cracks, dents, boils, or corrosion is recorded. Gaps at the quoin and miter points are measured. Dial gages are used to measure motion in the gate anchorage and wear between the gudgeon pin and its bushing. Downstream motion of the gates is monitored as three stages of static head are applied.

For steel sheet pile, each occurrence of misalignment, settlement, and cavity formation is logged. Interlock separation, cracks, holes, and dents are also noted as well as levels of corrosion.

The REMR Management Systems are being developed for a broad range of Civil Works structures and equipment. For navigation locks, systems have been completed for concrete lockwalls (McKay and Kao 1990), miter gates (Greimann, Stecker, and Rens 1990), steel sheet-pile walls, and steel sheet-pile mooring cells (Greimann and Stecker 1990).

Other systems currently under development include rubble breakwaters and jetties, sector lock gates, filling and emptying valves, tainter dam gates, stone and stone-filled-pile training dikes, steel sheet-pile/nonrubble breakwaters and jetties, and hydro power equipment.

Training sessions are planned for FY92 for the South Atlantic and Southwestern Divisions. The dates for these sessions have yet to be finalized.

For more information about the REMR Management Systems, contact Dave McKay at COM (217) 398-5487 or FTS 958-5487. District personnel interested in attending a training session should contact Dr. Tony Kao at (217) 398-5486.

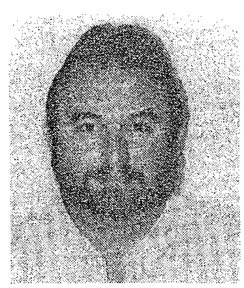
References

Kao, A. M. 1991. "Automating Maintenance and Repair - the REMR Management Systems for Civil Works Structures," *The REMR Bulletin*, Vol. 8, No. 1, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

McKay, D. T., and Kao, A. M. 1990. "LOCKWALL: A Microcomputer-Based Maintenance and Repair Management System for Concrete Navigation Lock Monoliths," Technical Report REMR-OM-10, the US Army Construction Engineering Research Laboratory, Champaign, IL.

Greimann, L., and Stecker, J. 1990. "REMR Management Systems - Navigation Structures, Management System for Miter Lock Gates," Technical Report REMR-OM-8, US Army Construction Engineering Research Laboratory, Champaign, IL.

Greimann, L., Stecker, J., and Rens, K. 1990. "Maintenance and Repair of Steel Sheet-Pile Structures," Technical Report REMR-OM-9, US Army Construction Engineering Research Laboratory, Champaign, IL.



Dave McKay is a research civil engineer in the Engineering and Materials Division of the US Army Construction Engineering Research Laboratory in Champaign, Illinois. He has a Bachelor of Science degree in Civil Engineering (Structures) and a Master of Science degree in Theoretical and Applied Mechanics, both from the University of Illinois. Dave spent 5 years in postgraduate studies in experimental stress analysis before joining the Corps in 1985 and has been working in the Operations Management Problem Area of the REMR Research Program since 1987.

Spillway Remediation at Saylorville Lake

by

Keith Haas, Glen A. Hotchkiss, and George Mech
US Army Engineer District, Rock Island

An erodible-rock spillway incorporated into a well-designed structure is a cost-effective use of natural materials. Modifications to prevent excessive erosion at the Saylorville Spillway will enable the structure to safely pass the spillway design flood (SDF).

The Saylorville Dam is located on the Des Moines River in Polk County, approximately 9 miles north of Des Moines, Iowa. The main dam is an earth-fill structure with an uncontrolled concrete spillway. The conservation pool for the reservoir is elevation 836 National Geodetic Vertical Datum (NGVD) 1929. It extends 19 miles upstream and provides a 5,950-acre lake. At full flood control pool (890 NGVD), the lake extends 54 miles above the dam and has a water surface area of 16,700 acres. The spillway design flood is 195,000 cubic feet per second (cfs).

The emergency spillway is a concrete ogee crest with 200 ft of paved apron. The apron slopes at 1 percent grade. The spillway is 430 ft wide with vertical walls extending 200 ft in length and 15 ft high. The remainder of the spillway channel is composed of a trapezoidal section, approximately 310 ft wide at the base.

Site geology

The bluffs flanking the Des Moines River valley have a foundation of Pennsylvanian age rocks capped by Quaternary age deposits of glacial till, alluvium, and loess. The bedrock consists of a repetitive layer-cake of sandstones, siltstones, shales, limestones, coals, and mudstones (Figure 1). The sandstone and limestones are relatively dense and erosion resistant. Fresh exposures of the siltstones and shales are also resistant; however, if subjected to repeated wetting and drying and freezing and thawing, they weather rapidly. A unit of siltstone/sandstone forms the channel floor, and a relatively flat grade of siltstone/sandstone exists for approximately 900 ft from the end of the paved concrete apron. During construction, a soil cap approximately 4 ft thick was placed over the siltstone and sandstone to protect the spillway from cycles of freezing and thawing.

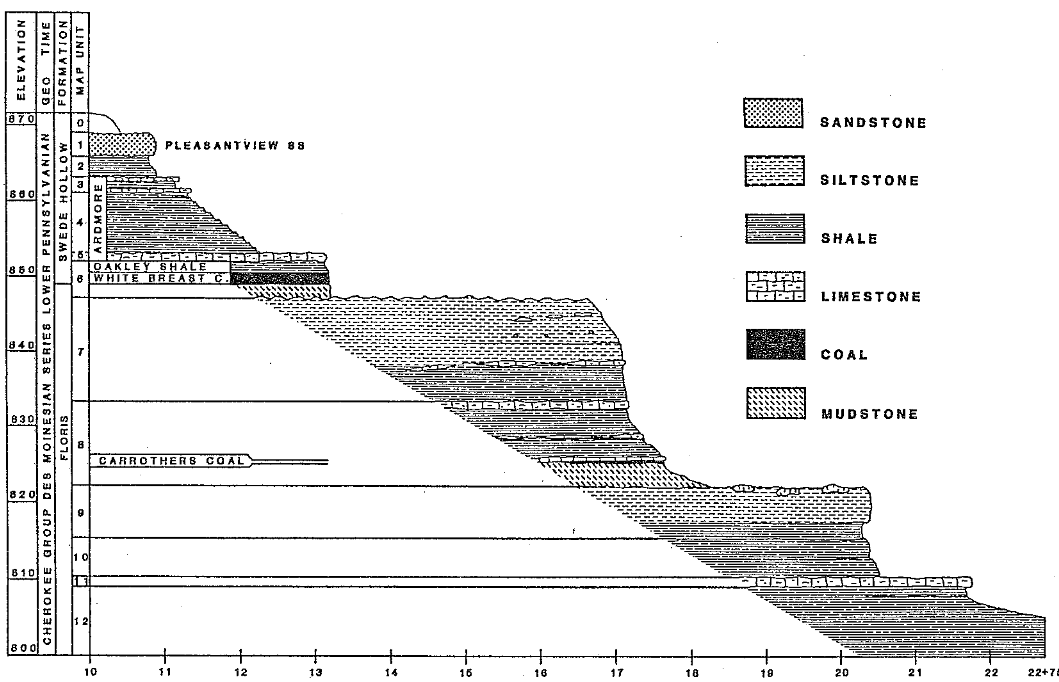


Figure 1. Saylorville Spillway geologic profile along centerline.

1984 overflow event

The dam became operational in 1977. In 1980, geologists and hydraulic engineers made predictions regarding the erodible nature of several rock formations and the erosion resistance of several other rock formations. Four years later the Saylorville Spillway experienced its first overtopping, which lasted 15 days. The pool reached a peak elevation of 889.2. The maximum flow over the spillway was approximately 17,000 cfs. The overtopping exposed various rock strata that had been covered during original construction. The exposed rock was uncovered approximately 1,100 to 2,000 ft downstream of the ogee crest where the hydraulic gradient forced supercritical flow and high velocities. The 4-ft soil cap with a good stand of grass was not eroded on the flat slope.

Survey profiles taken after the overflow showed that the amount of erosion which occurred through downcutting roughly equaled the 1980 estimate made for a flow of 40,000 cfs. The greatest amount of erosion occurred where anticipated; however, the depth of scour and downcutting exceeded these estimates. This unexpected depth of erosion was attributed to the relatively low tailwater elevation of 808.0 NGVD.

A supplement to the Saylorville Lake Periodic Inspection Report No. 7 was prepared in 1986. It addressed further geologic, geotechnical, and hydraulic data obtained from the 1984 overflow event. However, repairs were postponed until the results of studies on rock erosion in emergency spillway channels were made available. The studies were being done as part of the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program. (For additional information on these studies, see Cameron et al. (1986), Cameron et al. (1988a), Cameron et al. (1988b), and May (1989).)

Spillway modifications

In May 1988, a workshop sponsored by the Office of the Chief of Engineers (OCE), Waterways Experiment Station (WES), and the Rock Island District (NCR) met in Des Moines, Iowa. This workshop included a site visit to the Saylorville spillway, a presentation of studies conducted to date, and a planning session regarding possible repair alternatives for the Saylorville site. The general consensus of this group was that the spillway could be endangered by the possible erosion of rock produced by one event of an SDF. A major concern of the REMR group was headcutting, a progression of erosion back toward the spillway that could undermine the concrete structure. (The REMR group had conducted physical model studies at WES to document this headcutting phenomenon.)

In mid-1988, the District awarded a construction contract to remove the soil cap, which had been placed in the late 1970s. Upon removal of the cap, geologists from the Rock Island District mapped existing joint patterns and joint sets in the overlying capstone. The District then contracted with the Mobile District to mobilize onsite and take several cores in the spillway channel. Eight cores were taken with three cores extending 100 ft in depth. In December 1988, a ground-penetrating radar (GPR) survey of the surface caprock was performed. This survey was conducted in an attempt to correlate GPR data with caprock thicknesses actually observed in bore holes. The results of the GPR survey correlated rather well with existing borings in the spillway channel where the signal was not attenuated by interference.

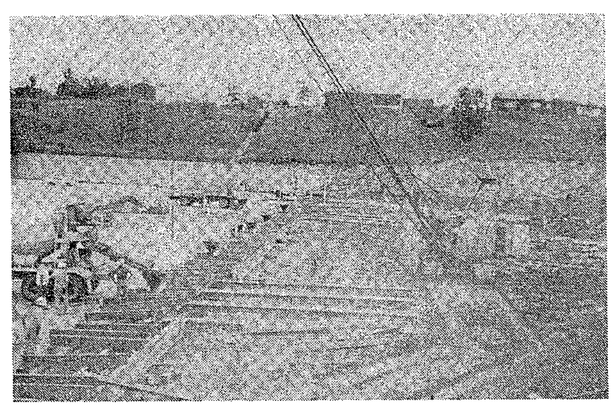


Figure 2. Installation of concrete cutoff wall.

After studying the reports on the integrity of the rock formations in the spillway channel, the District combined features from different alternatives to achieve an acceptable final remediation plan. Two major criteria were used to evaluate alternatives: the ability of the spillway structure and downstream spillway corridor to withstand floods up to the 100-year event without significant damage or maintenance expenditures and the ability of the spillway structure to safely pass an SDF without jeopardizing the integrity of the dam.

A critical line of defense to ensure that headcutting did not undermine the spillway was the installation of a concrete cutoff wall immediately downstream of the concrete apron (Figure 2). The wall is approximately 470 ft long, 35 ft high, and 3.5 ft thick. It is keyed into a dense siltstone layer. The cutoff trench was installed by presplitting the rock strata prior to excavation (Figure 3) and placing reinforcing steel mats in each face of the cutoff wall. The cutoff wall construction consisted of one monolithic concrete placement of approximately 1,750 cu yd of concrete.

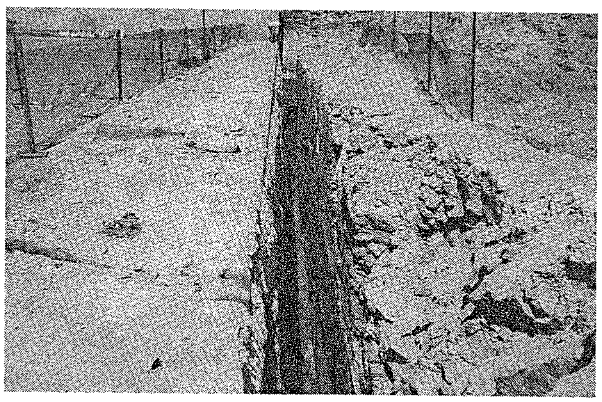


Figure 3. Excavated trench for installation of concrete cutoff wall.

A longer transition zone was provided between the rectangular concrete cross section to the trapezoidal section and included a 10-percent convergence to help eliminate the potential for standing waves, turbulence, and formation of eddies. Another line of defense was installed between 800 and 960 ft downstream of the ogee crest. A dense layer of siltstone blocks approximately 5 ft thick was anchored with epoxy-coated Number 8 rock anchors to underlying siltstone formations approximately 35 ft deep. One hundred and twenty anchors were installed on approximately 20-ft centers each direction. Following installation of rock anchors, the contractor replaced the soil cap 2 ft thick across the erodible rock spillway. The thickness of the soil cap was reduced to force a hydraulic jump to sweep out and move farther downstream of the spillway apron during overtopping.

The work was done during the summer of 1989 at a cost of approximately $660,000.

1991 overflow event

In June of 1991, the Des Moines River Basin experienced rainfall amounts of approximately 250 percent above normal. The reservoir reached a spillway crest elevation of 884.0 on June 6, 1991. A maximum pool elevation of 889.03 was achieved on June 9 and the pool receded to 884 June 15. Peak spillway flows of approximately 16,000 cfs were passed over the spillway at the peak reservoir elevation.

Following the overtopping event, geologists from the Rock Island District once again were sent to the Saylorville Spillway to identify and document erosional features and compare them with the 1984 overtopping of the spillway. Their investigation determined that although there were several isolated patches of soil cover which were eroded, the cover generally stayed in place. As the flow increased, the weathered rock was removed, and some material from the resistant beds was lost to undercutting. Large pieces of caprock that had been previously jointed but in place were displaced as they migrated downslope on the soft underlying shale (Figure 4). However, the caprock sandstone lost very little new material, and that which was lost was jointed and had been left uncovered. When the water receded, several of the rock units had advanced their headcutting upstream by a substantial amount, but there was little additional lateral erosion on any of the units.

Comparison of the 1991 overtopping to the 1984 overtopping shows that very little erosion of unweathered rock occurred during the 1991 overtopping with a similar type flow as was experienced in 1984. Performance of the spillway would likely have been better had the grass, planted in 1989 as a soil cover, had a few more years to root and spread.

With the modifications made in 1989, the spillway will safely pass an SDF of 195,000 cfs, but overtopping is expected to occur once in 25 years instead of once in 100 years as originally intended.

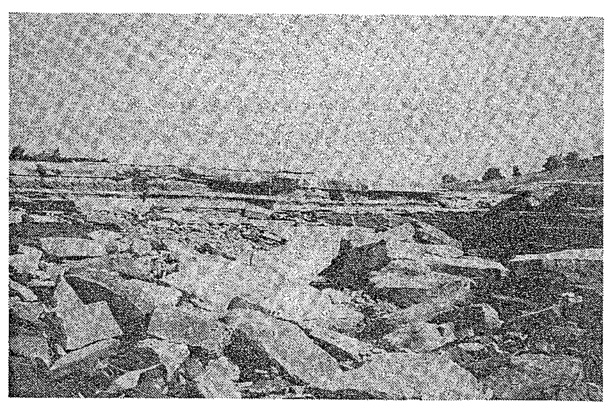


Figure 4. Blocks of caprock sandstone displaced in 1991 overflow event.

Spillway design change

When constructed, the project was authorized to have a flood control pool with a 100-year spillway crest elevation at 884.0 NGVD and a total capacity of 600,000 acre-ft. However, construction of the Big Creek remedial works resulted in a loss of 39,000 acre-ft of total storage at the full flood-pool elevation. In July 1965, OCE suggested a review of the project to provide for restoration of the lost storage. The recommendation was to increase the flood control storage to 602,000 acre-ft by raising the flood control pool from 884.0 to 890.0 NGVD. A total release rate (spillway plus outlet) of 21,000 cfs can be accomplished by operation of the gates in the outlet works until 889.0 NGVD is reached. Between elevation 889.0 NGVD and 890.0 NGVD, the gates of the conduit are gradually opened to increase the total release rate to 42,000 cfs. Above 890.0 NGVD (full flood-pool elevation), there is no regulation of the outlet works. With these changes, the spillway crest is reached with a 1 in 25-year reservoir pool frequency.

NCR is studying alternatives to modify the spillway structure to return it to the use for which it was originally intended. The spillway will then be reclassified from a limited-service spillway to an emergency spillway.

Documentation available

The District has documented the 1984 and 1991 spillway events as well as the repair contract in a series of videotapes. The geologic and hydraulic data are also contained in several design analysis reports and supplements to periodic inspection reports.

For further information, contact Keith Haas at (309) 788-6361, ext 6603; Glen Hotchkiss at (309) 788-6361, ext 6290; or George Mech at (309) 788-6361, ext 6288.

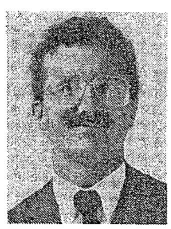
References

Cameron, Christopher P., Cato, Kerry D., McAneny, Colin C., and May, James H. 1986. "Geotechnical Aspects of Rock Erosion in Emergency Spillway Channels; Report 1," Technical Report REMR-GT-3, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Cameron, Christopher P., Patrick, David M., Cato, Kerry D., and May, James H. 1988a. "Geotechnical Aspects of Rock Erosion in Emergency Spillway Channels; Report 2, Analysis of Field and Laboratory Data," Technical Report REMR-GT-3, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Cameron, Christopher P., Patrick, David M., Bartholomew, Craig O., Hatheway, Allen W., and May, James H. 1988b. "Geotechnical Aspects of Rock Erosion in Emergency Spillway Channels; Report 3, Remediation," Technical Report REMR-GT-3, US Army Engineer Waterways Experiment Station, Vicksburg, MS .

May, James H. 1989. "Geotechnical Aspects of Rock Erosion in Emergency Spillway Channels; Report 4, Geologic and Hydrodynamic Controls on the Mechanics of Knickpoint Migration," Technical Report REMR-GT-3, US Army Engineer Waterways Experiment Station, Vicksburg, MS.



Keith Haas, a project engineer in the Engineering Division, Design Branch, Rock Island District, has been with the Corps since 1979. He received a Bachelor of Science degree in Civil Engineering from the University of Wisconsin at Platteville and a Master of Science degree in Civil Engineering from the University of Illinois. He is a registered Professional Engineer in the States of Illinois, Iowa, and Wisconsin.

Glen Hotchkiss is a geologist in the Geotechnical Branch, Engineering Division, Rock Island District. He received a degree in geology from Ball State University in 1969 and is a registered geologist in the States of Indiana and North Carolina. Glen has worked at Rock Island since 1983. Prior to joining the Rock Island District, he was an Active Army Engineer Officer.

George Mech is Chief of the Geotechnical Branch in the Rock Island District's Engineering Division. He has 17 years of experience with the Corps of Engineers, working in the New Orleans and Rock Island Districts and the European Division. George received a Bachelor of Science degree in Civil Engineering from Iowa State University and a Master of Science degree in Civil Engineering from Oklahoma State University and has attended the US Army Corps of Engineers-sponsored course in Concrete Materials at Purdue University. He is a registered Professional Engineer in the States of Iowa and Louisiana.

Epoxy Injection of Pier Stems at Mississippi River Dam No. 20

by

George J. Mech and Jerry Wickersham
US Army Engineer District, Rock Island

The Rock Island District has been systematically rehabilitating lock and dam facilities located within the District boundaries on the Illinois Waterway and Mississippi River. As part of this strategy, the wall surface of Lock 20 was replaced with conventional air-entrained concrete in 1986 (see *The REMR Bulletin*, Vol. 4, No. 4). Additional rehabilitation involving epoxy injection of 36 of the 42 dam bridge pier stems was performed in 1988-1989.

Background

Lock and Dam No. 20 is located 1 mile upstream of Canton, Missouri, at Upper Mississippi River mile 343.2. It is one of 29 locks and dams on the upper Mississippi River that operate as a system to provide a 9-ft navigation channel from St. Louis, Missouri, to Minneapolis, Minnesota.

The lock and control house are located on the Missouri shore. The main lock has a typical 110-ft-wide and 600-ft-long chamber. The maximum lift at the facility is 10 ft. The 2,144-ft-long dam consists of 40 tainter gates (6 of which are submersible) and 3 roller gates. The tainter gates are 40 ft wide and 20 ft high, and the roller gates are 60 ft wide and 20 ft high. Construction of this facility began in November 1933 and was completed in April 1936 at a cost of $6,152,000.

Problem

During the more-than-50 years of the structure's existence, severe climatic conditions, coupled with initiation of cracks and poor drainage at the top of piers, had contributed to the deterioration of the non-air-entrained concrete. It is believed the cracks were initiated by the inability of the expansion slots in the bridge girders to move freely with temperature variations. Broken bearing bolts found during periodic inspections of the dam support this hypothesis.

Additionally, the bridge bearings were recessed on many of the piers, resulting in poor drainage that allowed water to pond on top of the piers. This water then percolated down through the cracks into the pier stem. Cores taken from the structure at various times through the last 10 years were subjected to petrographic examination by the Missouri River Division Laboratory; these tests revealed the cause of the concrete deterioration to be from reaction to cyclic freezing and thawing and to alkali aggregate.

Investigations

In 1985, personnel from the Rock Island District's Geotechnical Branch conducted a concrete condition survey of the lock and dam. This survey recommended the removal and replacement of 15 of the 42 bridge pier stems. These severely deteriorated piers were 19 through 26, 31, 32, and 38 through 42 (Figures 1 and 2). The recommendation for the other 21 less deteriorated pier stems was to selectively remove portions of the piers and then replace these with conventional air-entrained concrete as well as improve the drainage of all the pier tops.

There was concern in the District about temporary support for the bridge and associated costs

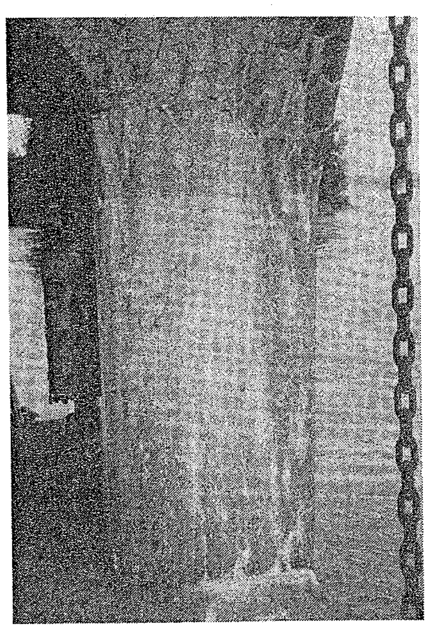


Figure 1. Upstream face of Pier 42 as seen during the concrete condition survey performed in 1985.

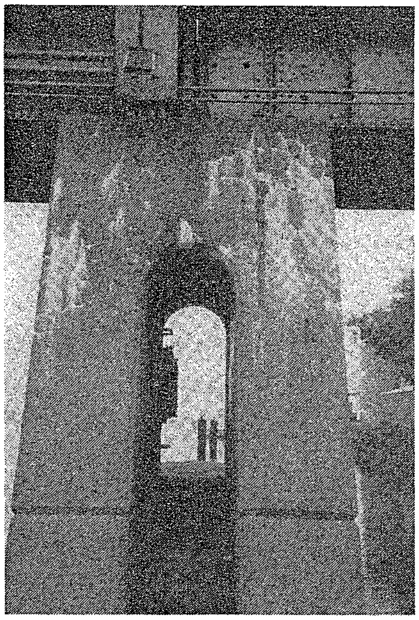


Figure 2. Downstream face of Pier 42 as seen during the concrete condition survey performed in 1985.

during the construction of the new piers. However, when the construction contract was awarded, the contractor decided to remove the bridge to facilitate the required work.

Proposed solution

In lieu of the proposed removal and replacement, the District proceeded to design an epoxy injection program, based on available information (a report on the epoxy injection of Pier 39, Dam 20, in 1982; and Technical Reports REMR-CS-6 and REMR-CS-21) in an attempt to save money but yet successfully rehabilitate the deteriorating structure. In June 1988, a contract that included epoxy injection of 36 piers was awarded. This contract also called for the removal of the top 1.5 ft of all the pier stems, replacement with conventional concrete (with improved drainage features), and installation of new bridge bearings.

Construction work

The first step in the epoxy injection procedure was to clean the surface of the pier with a power grinder. Holes for ports were drilled using a vacuum-attached swivel drill chuck, and ports were installed along the crack network (Figure 3). A water pressure test was then accomplished, and additional ports were set, based on the results. Water pressure tests were limited to a maximum of 40 pounds per square inch (psi).

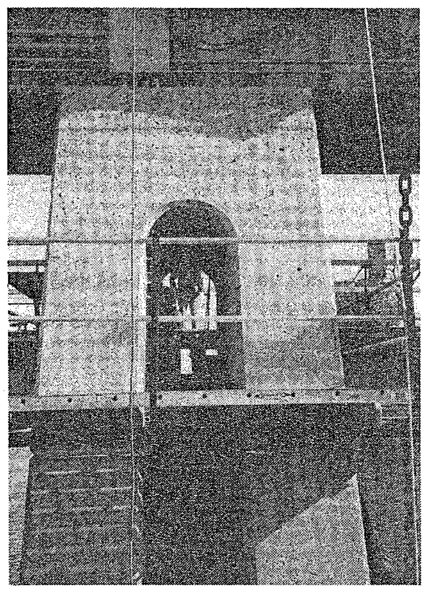


Figure 3. Downstream face of pier 42 with injection ports being located and installed. A total of 1,052 injection ports were used in Pier 42.

The surface was sealed with an epoxy gel (Sikadur 32) (Figure 4). Denepox 40, an ultra low-viscosity, two-component 100-percent solids epoxy resin insensitive to the presence of water, was injected through the ports into the pier. Injection started at the lowest point of the area on the pier and moved up (Figure 5). Injection pressures were limited in the specifications to a maximum of 160 psi.

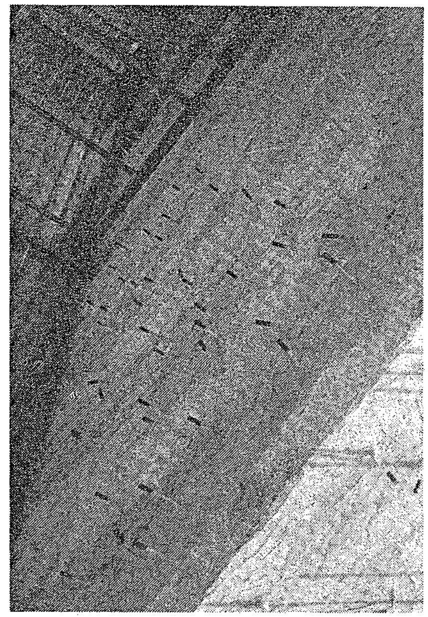


Figure 4. Typical arrangement of injection ports and epoxy gel surface seal on the side of the piers.

Contract bid unit prices for the epoxy injection work to be accomplished are given in Table 1.

Figure 5. Locating injection ports (denoted by an x) in the archway of the pier.

Quality assurance

Various techniques were employed to monitor the adequacy of the epoxy injection work. Preinjection and postinjection pulse velocities were determined. Concrete cores taken by the contractor were subjected to visual and petrographic examinations. Length of injected and noninjected cracks were determined by the petrographic analysis along with maximum depth of epoxy injected. Pulse velocities, splitting tensile strengths, and unconfined compressive strengths were determined on various portions of the cores.

Table 1. Contract Bid Unit Prices for Epoxy Injection

Work Item	*Unit*	*Price/Unit*
Mob and demob		
First 20	pier	$10,000.00
Over 20	pier	$500.00
Preparation and finishing		
First 20	pier	$1,500.00
Over 20	pier	$1,000.00
Injection ports & drilling		
Type A (surface)		
First 2,000	each	$25.00
Over 2,000	each	$15.00
Type B (6-in. depth)		
First 1,000	each	$15.00
Over 1,000	each	$11.00
Type C (12-in. depth)		
First 500	each	$25.00
Over 500	each	$20.00
Water pressure testing		
First 60	hour	$200.00
Over 60	hour	$92.00
Injected epoxy adhesive		
First 160	gallon	$300.00
Over 160	gallon	$258.00
Concrete cores		
First 50	l.f.	$150.00
Over 50	l.f.	$85.00

Summary of findings

The findings for the 13 pier stems injected and cored during the 1989 construction season include the following:

Differences between the preinjection and postinjection pulse velocities taken on the piers were negligible. If velocities were low, they remained low; alternately, if they were initially high, they remained high.

The cores obtained were 4 in. in diameter and typically 1.5 to 1.6 ft long. The total length of fractures identified along the outside surface ranged from a minimum of 9.4 to 27.7 lineal feet in the upper zone cores and 9.5 to 17.5 lineal feet in the lower zones cores.

The petrographic examinations determined that an average of 69 percent of the cracks in the upper zones of the cores were filled with epoxy. The typical range in the upper zone is from 0.00 to 0.75 ft (measured from the pier surface). Piers that exhibited less cracking had a greater percentage of fractures filled than those that exhibited more cracking. Cores from Piers 16 and 28 (less cracking) averaged 74 percent (low of 40 percent and high of 83 percent) of the fractures filled in the upper zone. Cores from 19, 22, and 31 (more cracking) averaged 61 percent (low of 34 percent and high of 82 percent) of the fractures filled in the upper zone.

Cores from Pier 42 (more cracking) averaged 89 percent of the fractures filled in the upper zone (low of 83 percent and high of 94 percent). Pier 42 was injected using 1,052 ports (approximately 1 for every 0.75 square feet of pier surface to be injected).

The petrographic examinations determined that an average of 31 percent of the cores in the lower zone (0.75 to 1.6 ft typically) of the cores were filled with epoxy. This ranged from a low of 0 percent to a high of 63 percent.

The average compressive strength of the epoxy-injected upper cores was 3,727 psi (with a range of 2,596 to 5,858 psi). The average compressive strength of cores taken in "sound" concrete (no epoxy) from the piers was 6,212 psi. The average splitting tensile strength of the epoxy-injected cores was 494 psi (with a range of 315 to 635 psi). The average pulse velocity of the epoxy-injected cores was 7,313 feet per second (fps). This falls in the range for poor quality concrete (7,000 to 10,000 fps).

The epoxy showed variable bonding strength ranging from well bonded in thick sections to poorly bonded where the epoxy was thin; that is, the crack was fine.

Re-evaluation and design change

After analysis of the data and evaluation of the effectiveness of the epoxy injection program, the District decided to abandon the epoxy injection method and opted to modify the contract to remove and replace the remaining badly cracked and deteriorated pier stems. Seven piers stems (31, 32, and 38 through 42) were removed and replaced. The contractor's proposal to do this work was $366,990 ($52,427 per pier stem). Since all work had already been accomplished on the other piers and the contractor had already replaced the service bridge (Figure 6), no additional work was required for these piers. They will be periodically inspected to evaluate the effectiveness of the injection process.

Overall, the average cost of epoxy injection was approximately $24,000 per pier. This cost would have been higher had the District

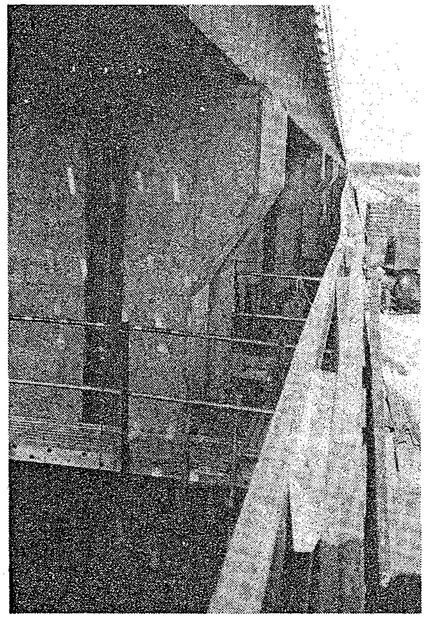


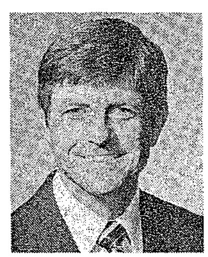
Figure 6. Contractor's scaffolding arrangement prior to the removal of the dam's service bridge.

allowed the contractor to use additional ports. The average cost to remove and replace the top 1.5 ft of concrete was approximately $17,000 per pier.

Although the epoxy injection was not as successful as the District had hoped, periodic investigation of the piers will aid in the determination of what percentage of voids filled is required to prevent further damage to the concrete from cycles of freezing and thawing. The addition of the new 1.5-ft air-entrained concrete along with new bridge bearings will add considerable longevity to the injected piers since a major source of free water has been eliminated.

For additional information, contact George Mech at (309) 788-6361, ext 6288, or Jerry Wickersham at (309) 788-6361, ext 6713.

George Mech is Chief of the Geotechnical Branch in the Rock Island District's Engineering Division. He has 17 years of experience with the Corps of Engineers, working in the New Orleans and Rock Island Districts and the European Division. He received a Bachelor of Science degree in Civil Engineering from Iowa State University and a Master of Science degree in Civil Engineering from Oklahoma State University, and he has attended the US Army Corps of Engineers course in Concrete Materials at Purdue University. George is a registered Professional Engineer in the States of Iowa and Louisiana.



Jerry Wickersham is a concrete technologist with the Geotechnical Branch of the Rock Island District. He has been with the Corps of Engineers since 1963. Jerry attended St. Ambrose College, the University of Iowa, and Purdue University.

Spring REMR-II Field Review Group Meeting

The third REMR-II Field Review Group (FRG) Meeting has been scheduled for 19-21 May 1992. Hosted by the North Central Division, the meeting will be held in the Rock Island/Davenport area. Approximately 2 days will be allowed for the Program Review (19-20 May), and a field trip to a repair or rehabilitation site is being planned for the morning of May 21.

For additional information on the meeting location and hotel accommodations, please contact Lee Byrne, Technology Transfer Specialist, at (601) 634-2587 or write U.S. Army Engineer Waterways Experiment Station, ATTN: CEWES-SC-A/TTS (Lee Byrne), 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

FY92 Program Documentation

A limited number of copies of the *FY92 Program Documentation* are available to those who are interested in learning more about REMR-II work units. Included in the report are the objectives of each study, milestones, principal investigators, and points of contact for more information. If interested in receiving a copy of this documentation, contact Lee Byrne, Technology Transfer Specialist, by calling (601) 634-2587 or by writing U.S. Army Engineer Waterways Experiment Station, ATTN; CEWES-SC-A/TTS (Lee Byrne), 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

Supplement 5 to *REMR Notebook*

Supplement 5 to the *REMR Notebook* is scheduled for distribution in March. The *Notebook* presently contains two volumes of technical notes and one volume of material data sheets. Entries are provided for each of the seven problem areas addressed by the REMR Research Program: Concrete and Steel, Geotechnical, Hydraulics, Coastal, Electrical and Mechanical, Environmental Impacts, and Operations Management. Supplements and revisions to the *Notebook* are made annually.

Readers who are interested in subscribing to the notebook should call Lee Byrne, Technology Transfer Specialist, at (601) 634-2587 or write to U.S. Army Engineer Waterways Experiment Station, ATTN: CEWES-SC-A/TTS (Lee Byrne), 3909 Halls Ferry Road, Vicksburg, MS 39180-6199.

Levee Rehabilitation

The Geotechnical Laboratory of the US Army Engineer Waterways Experiment Station conducted a REMR-II Workshop on Levee Rehabilitation on March 17 in Vicksburg, MS. The purpose of the workshop was to foster the interchange of ideas within the Corps of Engineers regarding innovative methods for levee rehabilitation. Topics on the agenda included "Seismic Assessment of Pajaro and San Lorenzo River Levees After Loma Prieta Earthquake," "Use of CONDOR® SS to Stabilize Levee Embankments," "Double Lime Application for Levee Stabilization," "Use of Geogrids for Levee Slope Stability Problems," "Lime Stabilization and Rock-Fill Trenches," "Lime-Fly Ash Injection of Levees," "Levee Construction on Soft Soils Using High-Strength Geotextiles," and "Use of Soil Nailing for Slope Repair." POC: Edward B. Perry, FTS 542-2670 or COM (601) 634-2670.

Ground-Penetrating Radar Equipment and Applications

A Government User's Workshop on Ground-Penetrating Radar (GPR) Equipment and Applications was held at the US Army Engineer Waterways Experiment Station on March 26 and 27. The objective of the workshop was to provide a forum for government users of GPR to be introduced to state-of-the-art and emerging technology in GPR equipment, consider current applications of GPR, and discuss limitations and areas of required research. Included were presentations by private industry, government agencies, and academia. Areas of application included archeology, hazardous and toxic waste site characterization, mine and unexploded ordnance detection, hydrogeology, ground-water exploration, subsurface contaminant mapping and characterization, cavity and tunnel detection, and geotechnical studies (dams, pavements, levees, and buildings). POC: Michael K. Sharp (601) 634-3787 or Dwain K. Butler (601) 634-2127.

The Condition Index (CI) serves as the heart of the REMR Management Systems for Civil Works structures. It captures a "snapshot" of a structure's condition and provides the means to compare the condition of similar structures on a global basis. A practiced inspection team of two or more persons can perform a complete inspection in less than 2 days. This issue includes a description of the Corps-sponsored, 5-day training session that introduces participants to CI inspection procedures for concrete lockwalls, miter gates, and steel sheet-pile walls and mooring cells.

The REMR Bulletin is published in accordance with AR 25-30 as one of the information exchange functions of the Corps of Engineers. It is primarily intended to be a forum whereby information on repair, evaluation, maintenance, and rehabilitation work done or managed by Corps field offices can be rapidly and widely disseminated to other Corps offices, other US Government agencies, and the engineering community in general. Contribution of articles, news, reviews, notices, and other pertinent types of information are solicited from all sources and will be considered for publication so long as they are relevant to REMR activities. Special consideration will be given to reports of Corps field experience in repair and maintenance of civil works projects. In considering the application of technology described herein, the reader should note that the purpose of *The REMR Bulletin* is information exchange and not the promulgation of Corps policy; thus guidance on recommended practice in any given area should be sought through appropriate channels or in other documents. The contents of this bulletin are not to be used for advertising, or promotional purposes, nor are they to be published without proper credits. Any copyright material released to and used in *The REMR Bulletin* retains its copyright protection, and cannot be reproduced without permission of copyright holder. Citation of trade names does not constitute an official endorsement or aproval of the use of such commercial products. *The REMR Bulletin* will be issued on an irregular basis as dictated by the quantity and importance of information available for dissemination. Communications are welcomed and should be made by writing US Army Engineer Waterways Experiment Station, ATTN: Lee Byrne (CEWES-SC-A), 3909 Halls Ferry Road, Vicksburg, MS 39180-6199, or calling 601-634-2587.

ROBERT W. WHALIN, PhD, PE
Director

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
3909 HALLS FERRY ROAD
VICKSBURG, MISSISSIPPI 39180-6199

OFFICIAL BUSINESS
CEWES-SC-A

Vicksburg, MS
Permit No. 85